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# LATERAL STRENGTH OF URM PIERS: COMPARISON BETWEEN CODIFIED CRITERIA AND IN-PLANE TEST RESULTS

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**Abstract.** *The lateral resistance represents one of the most significant wall parameters to be used in the seismic analyses for the design/assessment of masonry buildings. In this article, an investigation on in-plane lateral strength of URM piers has been proposed thorough a comparison between the results from codified criteria and the outcomes of several experimental in-plane cyclic tests on masonry walls. In this context, a new database collecting the results of in-plane cyclic tests on unreinforced masonry piers, carried out within different research projects, has been developed. The database consists of walls with bricks and blocks with different masonry materials (clay, lightweight aerated concrete, AAC, calcium silicate), bed-and head-joint typologies, dimensions, boundary conditions, vertical applied loads and horizontal loading history. This source of information of consistent and reliable test results represents a necessary step into the process of definition of shared rules in the European context.*

## 1 INTRODUCTION

In this paper, a preliminary investigation on the in-plane lateral strength of unreinforced masonry walls is proposed, since it represents one of the main parameter that may influence and govern the lateral response of URM buildings under seismic excitation and thus that has to be considered in the global seismic analyses for the design/assessment of masonry buildings.

Since a complete understanding of all the involved phenomena is still lacking, in particular regarding the evaluation of the values of lateral strength and the identification of the effective failure modes, a statistically significant database, that assembles essential information and experimental results of in-plane cyclic tests on unreinforced masonry walls, may provide a useful tool to improve the current analytical models and, possibly, to review code recommendations.

Examples of unified databases with results of in-plane tests on masonry walls, already available in literature, are for instance the ones recently described by Augenti et al. [1] and Gams et al. [2], or the database about stone masonry proposed by Vanin et al. [3].

A new database collecting the results of in-plane cyclic tests on unreinforced masonry piers, carried out mainly in Europe within different research projects and found in literature, is also presented. Several masonry materials (with bricks and blocks), bed- and head-joint typologies, specimen dimensions, boundary conditions, vertical applied loads, horizontal loading histories and failure modes are included.

The development of such a reliable source of consistent information about experimental results represents a required step for the definition of shared rules in the European context, with particular reference to the definition of specific performance limit states and related capacity models for the in-plane seismic response of structural masonry walls.

## 2 ORGANIZATION AND CONTENTS OF THE DATABASE

All the considered in-plane cyclic shear tests are performed with an initial application of a vertical load and, consequently, through a cyclically acting horizontal load applied at the upper part of the wall. The horizontal action is generally applied in the form of programmed displacements, cyclically imposed in both directions with step-wise increased amplitudes up to ultimate conditions of the specimens; at each displacement amplitude, the loading is repeated two or three times (this latter option for the majority of the tests). In addition, in the majority of experimental campaigns, tests of characterization on units, mortar and masonry are also performed and thus the available results are reported in the database.

The database is organized in eight sections with seventy-one columns of data, in addition to the first containing the sequential number of the specimens; the eight sections, whose number is reported in brackets, regard general information and reference [I], information on masonry type, units and mortar [II], information on masonry walls [III], test conditions [IV], calculated lateral resistances [V], experimental results through cyclic tests [VI], parameters of the bilinear curves [VII] and drift capacities [VIII].

A summary of all the data and parameters included in the different sections of the database is reported in Albanesi et al. [4], while all the considered sources, reported in the first section of the database, are listed in this paper in the references [5-27].

A total of 188 piers form the complete list, including 101 hollow clay with vertical perforation (HC), 26 solid unit autoclaved aerated concrete (AAC), 18 solid unit calcium-silicate (CS), 11 lightweight aggregate concrete with vertical perforation (LAC), 30 solid clay brick (SB-C) and 2 calcium-silicate brick (SB-CS) masonry piers, as reported in section [II] of the database. Figure 1 shows the composition of the database in terms of the masonry material of the tested specimens.

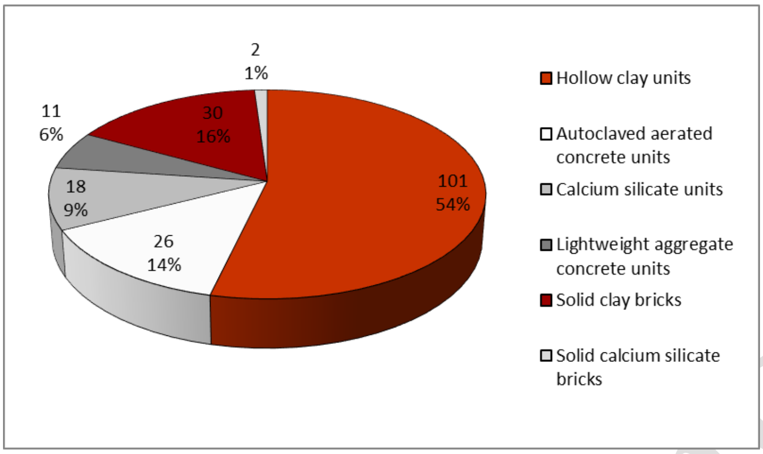



Figure 1: Masonry materials of the specimens included in the database.

The same section of the database, dedicated to information on units and mortar, reports information also about bed- and head-joints. Masonry walls with blocks are characterized by general purpose (GP) or thin layer (TL) mortar bed-joints and different types of head-joints, such as completely filled (F), filled with thin layer mortar (TF), filled in the pocket (MP), unfilled with plain (U) or tongue and groove (UTG) units. The clay and calcium-silicate solid brick masonry is instead realized only with general-purpose mortar and completely filled joints. Figure 2 shows the head- and the bed-joint typologies for the tested masonry walls.

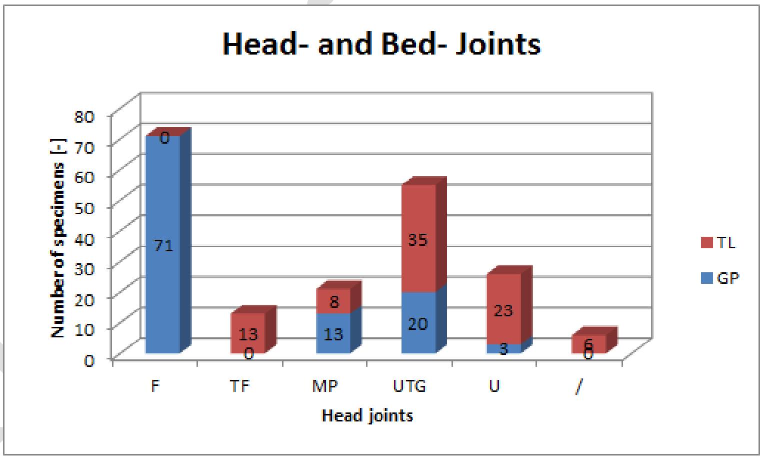



Figure 2: Head- and bed-joint typologies of the specimens included in the database.

The height of the piers, reported in section [III] of the database among other information about the masonry specimens, ranges from 1.17 m to 3.00 m, since only tests on walls with height larger than 1.15 m have been included in the database, while the majority are included in the interval between 2.25 and 2.50 m, as reported in Figure 3.

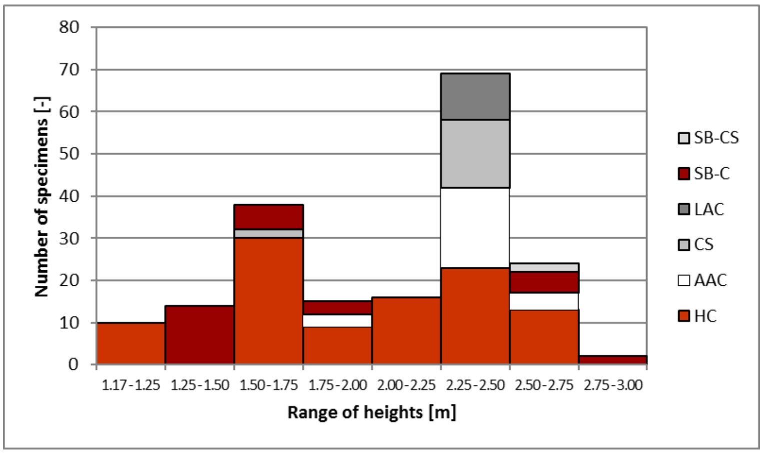



Figure 3: Number of specimens at given piers' height intervals.

The fourth section of the database reports the main information about the test conditions applied to each specimen, such as the boundary conditions or the vertical load. Regarding the static scheme, almost all of the tests are conducted with “Double Fixed” (no rotation of the top beam, “DF”) or “Cantilever” (free rotation of the top beam, “C”) boundary conditions, with exception of few specimens that are tested with intermediate conditions (“O”), as reported in Figure 4.

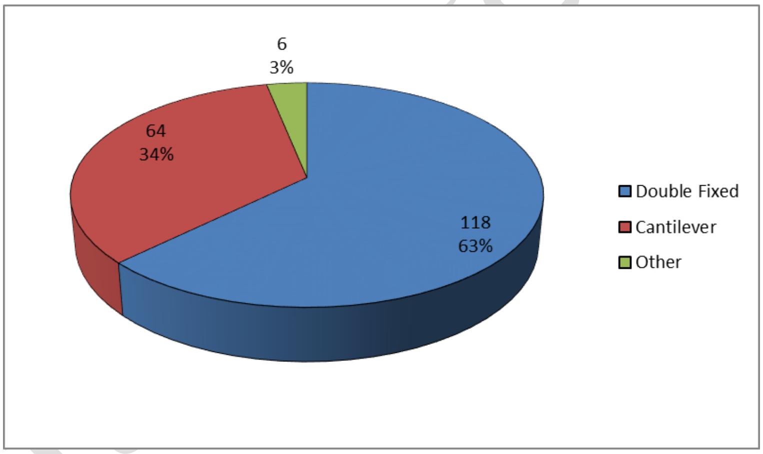



Figure 4: Composition of the database in terms of test boundary conditions.

As regards the vertical load applied to the specimen during the test, it is interesting to evaluate the ratio $\sigma_v/f$, between the resulting compression stress on the horizontal cross section of the wall and the compressive strength of the masonry. Figure 5 reports the number of specimens at given intervals of $\sigma_v/f$, showing that these ratios range from 2% to 41%, with the majority included between 2.5% and 22.5% and with the maximum value between 5% and 10%.

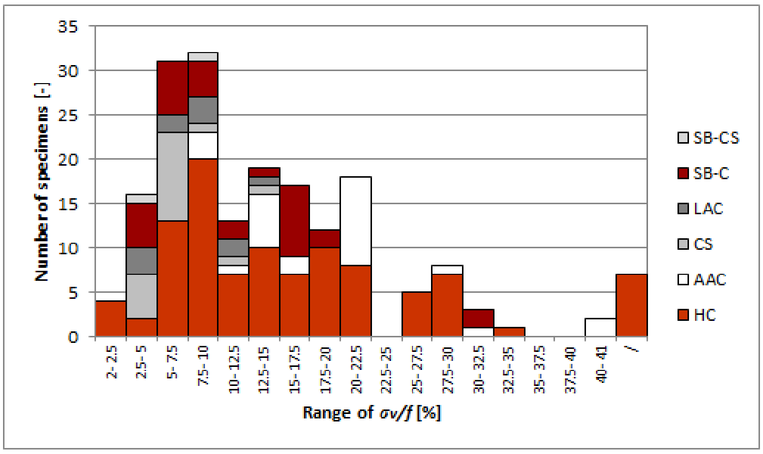



Figure 5: Number of the specimens at given intervals of $\sigma_v/f$; "\" indicates the cases when no compression strength was provided or evaluated.

The fifth section contains the calculation of the shear resistances of the specimens, starting from their geometrical and mechanical properties and according to the different code formulations proposed in Eurocode 6 (EC6 [28]) and in the Italian norms for construction (NTC2008 [29]), as reported in section 3.1 of this paper.

The failure modes actually obtained in the tests are instead reported in section [VI] of the database and cover different mechanisms, such as flexural/rocking (F), pure shear (S) with diagonal or step-wise cracking involving the joints and the units, sliding (SL) at the ends of the piers, "gaping" (G) with stepped cracking, and hybrid modes (H) with the occurrence of two different failure modes.

The failure modes identified in the original reports and papers have been carefully rechecked, for all the tests, according to the damage pattern of the specimens (evaluated from the available documentation), the type of hysteretic curves and the values of the maximum attained displacement; sometimes, this interpretation has led to improve the original evaluation of the mechanisms stated in the original sources, in particular in the case of "hybrid" mechanisms, where the main involved modes have been now explicitly specified (for example, "H-FS" defines an hybrid mode with the occurrence of flexural and shear mechanisms).

The sixth section contains also all the experimental results, distinguished for positive ($^{+}$) and negative ($^{-}$) direction, such as the peak lateral force $V_{max,exp}^{+/-}$ obtained during the test and the corresponding displacements $\delta_{Vmax,exp}^{+/-}$, the positive and negative maximum displacements of the last fully completed cycles $\delta_{max,f}^{+/-}$ (after two or three cycles, depending by the loading history) and the maximum displacements attained in the test in both directions $\delta_{max}^{+/-}$, independently by the full completion of all the cycles.

Section [VII] of the database is dedicated to the interpretation of the in-plane experimental response of the masonry walls; a common approach to evaluate the related seismic parameters is to idealize the cyclic envelope of the hysteresis loops by means of a bilinear curve. For all the tests, the approach described in Frumento et al. [12], summarized in Figure 6, has consistently been used and the obtained parameters are included in this section of the database.

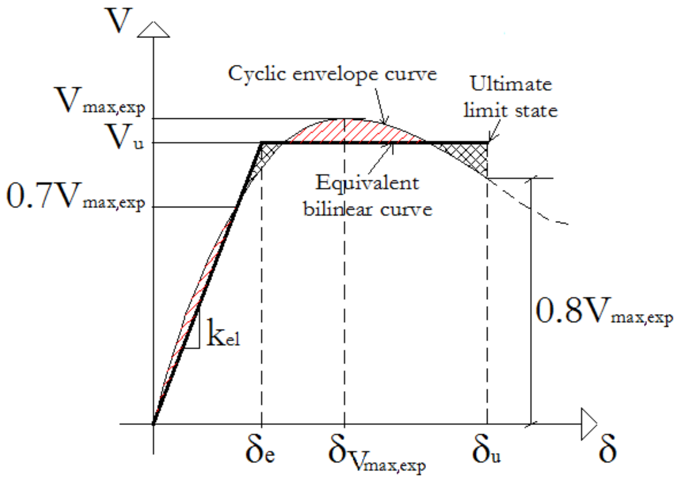



Figure 6: Idealization of the cyclic response: evaluation of the bi-linear curve from the hysteresis envelope.

Finally, the eighth section summarizes the main drift capacities identified for each specimen, calculated dividing the horizontal displacements $\delta$ obtained from the tests by the height $h$ of the specimens. With the exception of the drift at first crack $\theta_{cr}$, the other relevant drift values (at the elastic limit $\theta_e$, at peak force $\theta_{Vmax}$, at 0.8 times the peak force $\theta_u$, at the maximum displacement of the last fully completed cycle $\theta_{max,f}$, and at the maximum attained displacement $\theta_{max}$) are estimated taking the minimum displacement between the positive and the negative directions.

## 3 PRELIMINARY ANALYSIS OF THE RESULTS

In this section, a preliminary investigation on the in-plane lateral strength and failure modes of the walls is proposed; it is important to point out that only specimens with heights $h$ larger or equal to 1.50 m and more than 7 courses have been considered in the following results, in order to exclude the issues of “size effect”. In fact, wall specimens with very few courses of masonry units or very low heights can be subjected to higher influence of the boundary conditions (i.e. the confinement provided by the top and the bottom reinforced concrete or steel spreader beams) that can condition the results of the cyclic tests. Therefore, the sample turns out to be limited to 135 piers (62 hollow clay, 26 aerated autoclaved concrete, 18 calcium silicate, 11 lightweight aggregate, 16 solid clay brick and 2 calcium-silicate solid brick masonry), out of the 188 of the original database.

### 3.1 Lateral strength

The lateral strength corresponding to flexural failure $V_{flex}$ has been calculated as $M_u/h_0$, where the resistant bending moment $M_u$ of unreinforced masonry walls has been evaluated with the following expression, included in the Italian norms for constructions (NTC2008 [29]):

$$M_u = \frac{l^2 \cdot t \cdot \sigma_v}{2} \cdot \left(1 - \frac{\sigma_v}{0.85 \cdot f_d}\right) \tag{1}$$

where the masonry is assumed subjected to eccentric compression, with a rectangular stress diagram having a value of ultimate compression equal to $0.85 \cdot f_d$ ($f_d$ is the vertical compression strength of the masonry, taken as the mean value of the vertical compression strength), $l$ is the length, $t$ is the thickness of the wall and $\sigma_v$ is the mean compression stress on the section of the panel (calculated as $\sigma_v = N/(l \cdot t)$, in which $N$ is the vertical load).

Regarding the shear failure, the corresponding lateral strength has been calculated as the minimum value between $V_{shear}$ and $V_{shear,lim}$ evaluated with the following expressions derived from EC6:

$$V_{shear,i} = f_{vd} \cdot t \cdot l' \tag{2}$$

$$f_{vd} = f_{v0} + 0.4 \cdot \sigma_d \quad \text{for filled head-joints} \tag{3}$$

$$f_{vd} = 0.5 \cdot f_{v0} + 0.4 \cdot \sigma_d \quad \text{for unfilled head-joints} \tag{4}$$

$$V_{shear,lim,i} = f_{v,lim} \cdot t \cdot l' \tag{5}$$

$$f_{v,lim} = \frac{0.065}{0.8} \cdot f_b \quad \text{for filled head-joints} \tag{6}$$

$$f_{v,lim} = \frac{0.045}{0.8} \cdot f_b \quad \text{for unfilled head-joints} \tag{7}$$

where $f_{vd}$ represents the shear strength of masonry, $t$ is the thickness of the wall, $f_{v0}$ is the mean value of the initial shear strength, $\sigma_d$ is the mean compression stress on the portion of the wall subjected to compression (calculated as $\sigma_d = N/(t \cdot l')$, in which $N$ is the vertical load) and $f_b$ is the normalized vertical compression strength of the units. A coefficient equal to 1/0.8=1.25 has been applied to obtain a "mean" value of $f_{v,lim}$ from the characteristic value of $f_{vk,lim}$, in accordance with EN 1052-3 [30], that recommends a value of 0.8 between the mean and characteristic shear strength. Moreover, the expressions for the calculation of $V_{shear,i}$ and $V_{shear,lim,i}$ have been limited to an upper bound for the case of wall sections entirely subjected to compression, whereas $V_{shear,i}$ has also been limited to a lower value which corresponds to the minimum contribution of shear strength due to the masonry friction only.

The minimum value between $V_{flex}$, $V_{shear}$ and $V_{shear,lim}$ represents the predicted lateral shear resistance of the walls ($V_{pred}$).

Figure 7 a) and b) show a comparison between the values of the experimentally measured lateral resistance (considering the maximum between positive and negative direction) and the strength predicted by the codified expressions reported above, with the indication of the main experimental failure mechanisms.

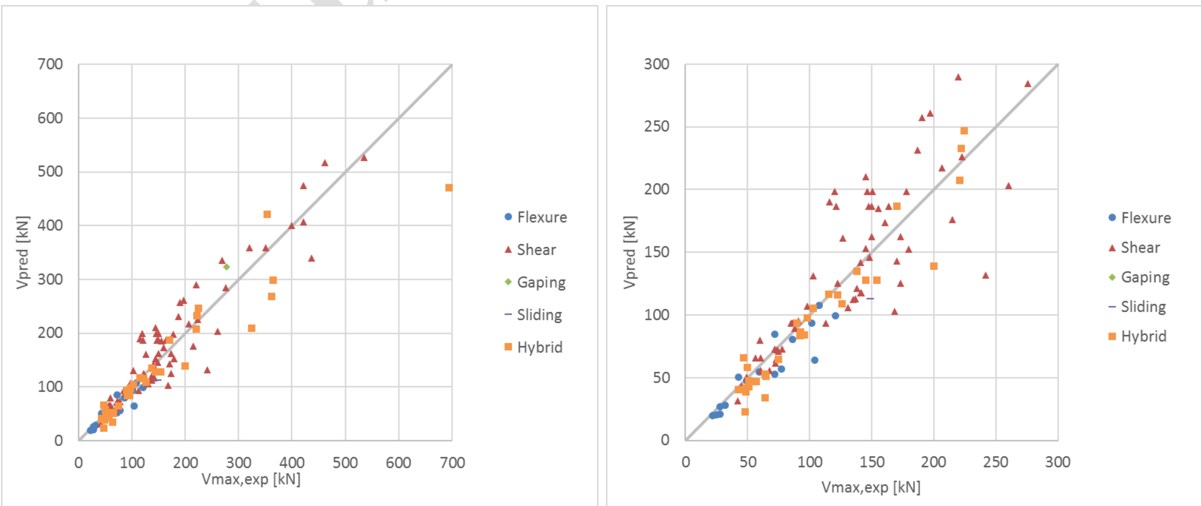



Figure 7: Correlation between experimental and predicted strength (a) and zoom up to forces of 300 KN (b).

Considering the estimation of the accuracy in terms of ratio between the predicted and the experimental values ($V_{pred}/V_{max,exp}$), reported in Figure 8, it is possible to notice that some of the specimens show a poor prediction with a non-negligible over or under estimation. The accuracy of the analytical strength prediction does not appear to be substantially influenced by the different effective failure modes, although it is clear that shear failures are more affected by over estimation compared to flexural and hybrid failures, which instead show much more cases of under estimation, thus resulting generally more safe-sided.

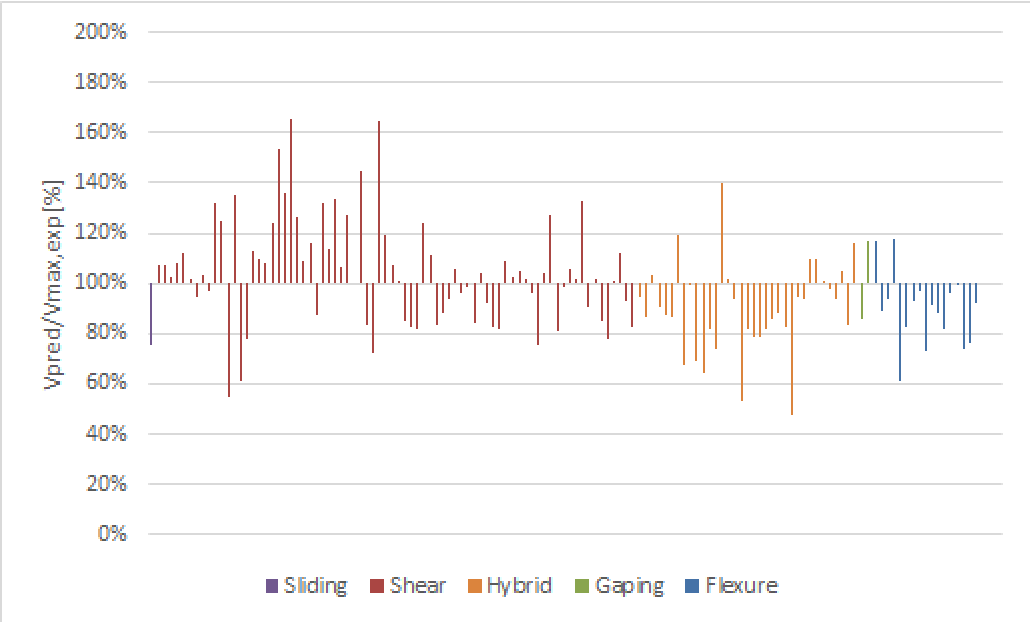
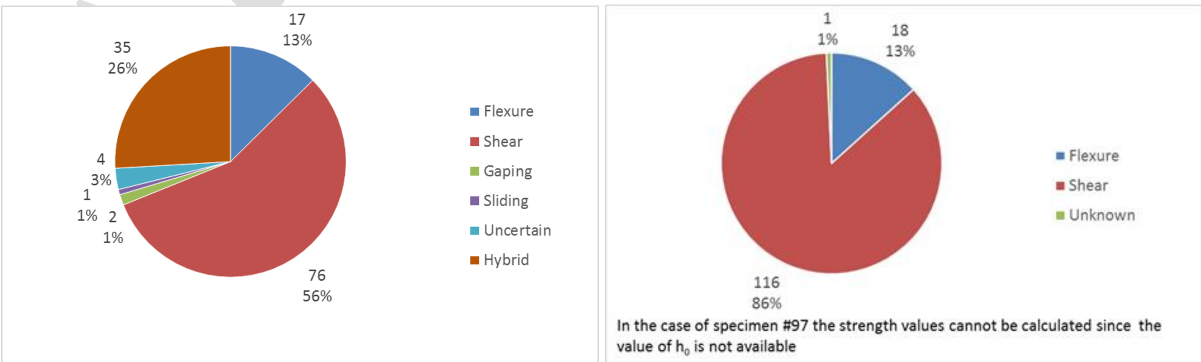



Figure 8: Ratios between the predicted and the experimental values of lateral resistance.

### 3.2 Failure modes

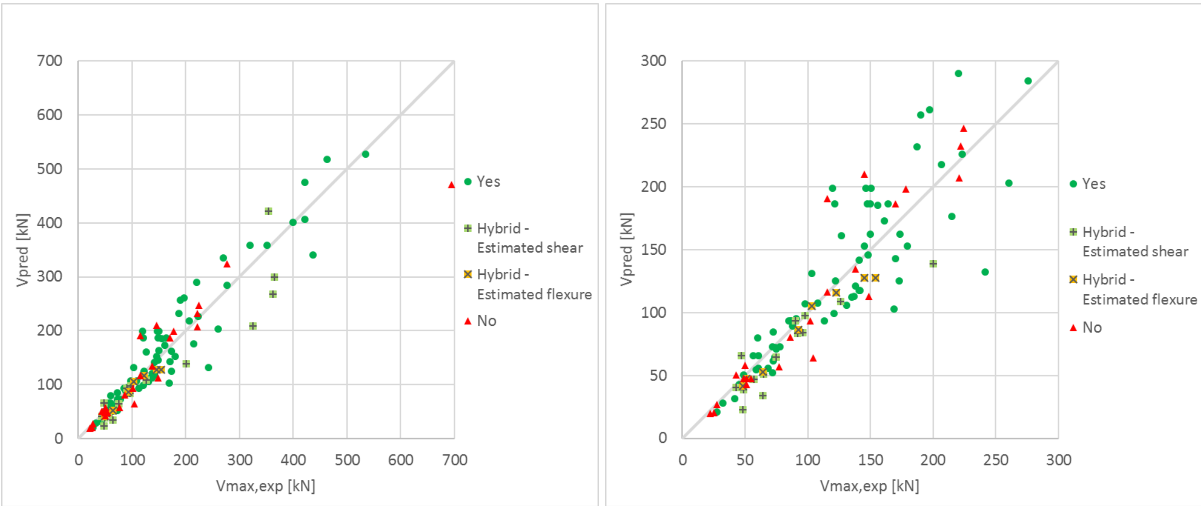
The lower calculated value between the flexural and the shear strength provides also the analytical identification of the failure mode that, therefore, could be by flexure (“F”) or by shear (“S”). Figure 9 a) illustrates the distribution of the failure modes derived from the interpretation of the experimental findings on the considered specimens, whereas Figure 9 b) shows the distribution of the predicted failure modes, by flexure or by shear, evaluated analytically.




a) b)

Figure 9: Distribution of the experimental (a) and predicted (b) failure modes.

Finally, Figure 10 a) and b) report once again the correlation between the experimental and the predicted strength, but representing with green dots the situation when the expected failure mode according to the code formulations is confirmed by the experimental results, and with red triangles when it is not; if the experimental test has provided a “hybrid” mode involving the predicted estimated mode, the markers are light green if the analytical estimation is for shear (and therefore someway safe-sided), otherwise the markers are orange if the estimation is for flexure.




a) b)

Figure 10: Correlation between experimental and predicted strength in terms of accuracy of prediction of failure modes (a) and zoom up to forces of 300 KN (b).

It appears clearly that the accuracy in the prediction of the failure modes is not dependent by a good estimation of the lateral strength value, as indicated in Figure 10 a) and b) by the several red triangles lying close to the grey line representing the perfect match between the effective and the estimated resistance.

Moreover, looking at Figure 11, the analytical estimation of the failure modes matches the effective experimental mechanisms for 61% of the specimens but, for 19% of the walls, the estimation is incorrect and increases up to 25% including the cases with “hybrid” modes involving also brittle mechanisms estimated, conversely, as pure flexural.

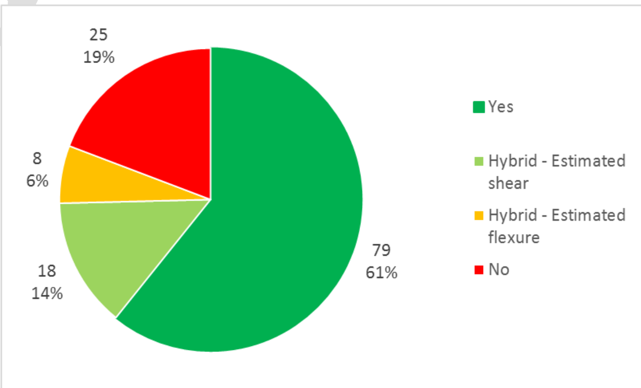



Figure 11: Number/percentage of specimens that match the failure modes.

## 4 CONCLUSIONS

In this paper, a preliminary investigation on the in-plane lateral strength of unreinforced masonry walls, through a comparison between the results of the codified strength criteria and the experimental outcomes based on available in-plane cyclic tests, is proposed.

A new database that gathers the information and the results of 188 in-plane cyclic tests carried out on unreinforced masonry piers with bricks and blocks has been developed. The considered specimens are characterized by different materials and typologies, dimensions, boundary conditions, vertical applied loads and horizontal loading history, and the failure modes obtained in the tests cover a wide range of cases, from flexural/rocking to pure shear and hybrid modes with the occurrence of two different failure modes.

It is important to underline that at this stage the largest effort has been devoted to the preparation of the database, aiming at the improvement of the understanding of the in-plane response of URM walls; the aspects here discussed concerning the lateral strength and the failure modes only represent an example of the considerations that can be carried out starting from these results.

For the proposed interpretation of the test results, only specimens with heights $h \geq 1.50$ m and with more than 7 courses have been considered, in order to avoid "size effect" issues that can condition the results of the cyclic tests.

The experimental lateral strength of the tested piers has been compared with the expressions provided by Eurocode 6 (EC6 [28]) and by the Italian norms for construction (NTC2008 [29]). The comparison between the experimental and the predicted lateral strength has shown a reasonable prediction for a significant number of specimens, but with some examples of over estimation of the effective lateral strength, especially in case of shear failures.

In addition, the failure modes have been identified analytically and then compared with the actual test results: the analytical estimation of the failure modes (limited by the code expressions to pure shear or flexural mechanism only) has led to incorrect results in about 20% of the specimens, up to 25% including the cases with "hybrid" modes estimated as pure flexural.

Therefore, the improvement of the current codified strength expressions and the development of new ones, that may consider all possible mechanisms and thus allow a better identification of the effective failure modes, is envisaged. Actually, on some typologies, more accurate strength criteria have already been proposed in past studies as, for example, by Magenes and Calvi [31] for brick masonry walls, but for other cases further investigation is still required.

## 5 ACKNOWLEDGMENTS

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